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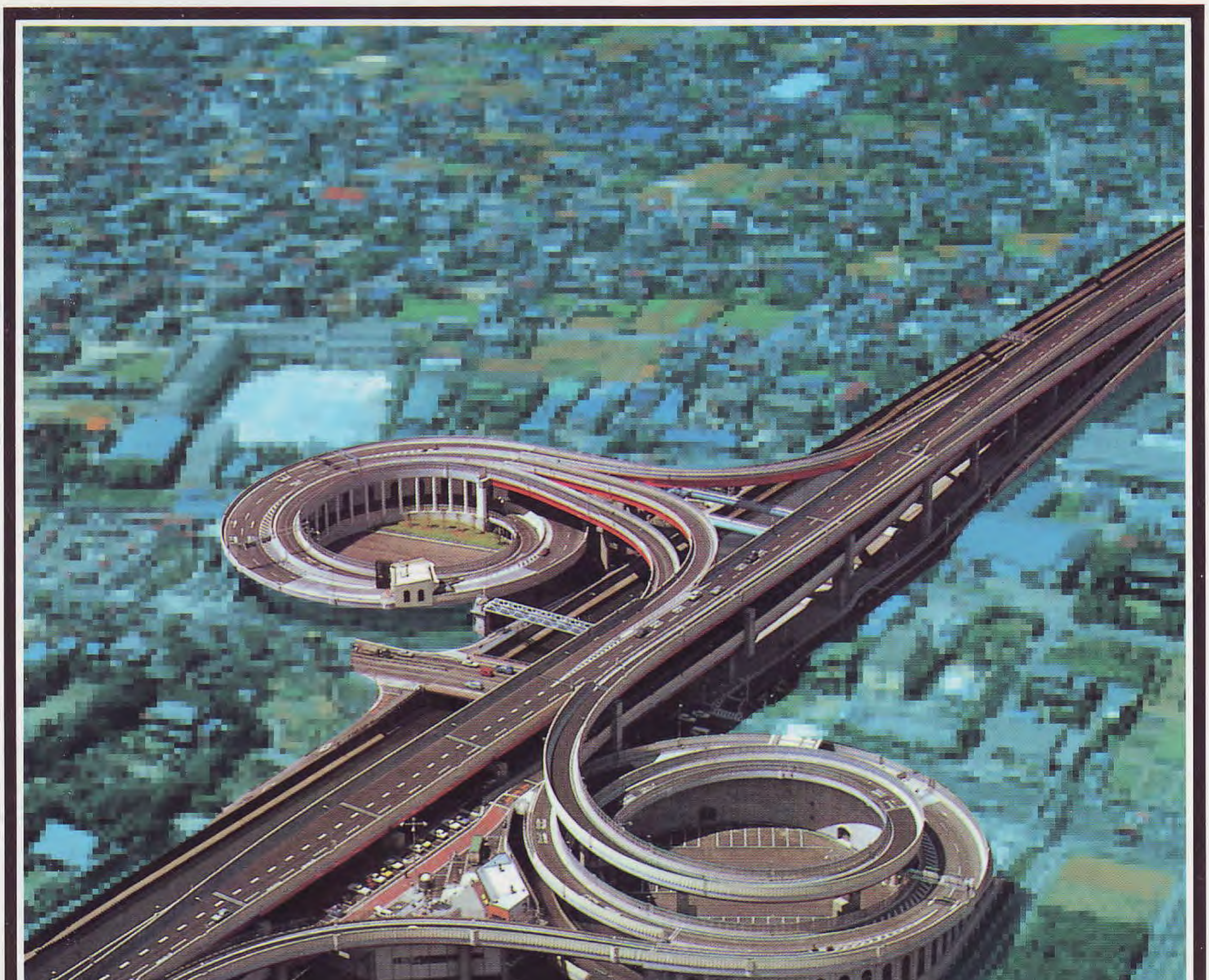

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COVER: Spiral ramps on Metropolitan Expressway, Japan.

The theme for this issue of the REAAA Journal is **LOW COST ROAD CONSTRUCTION AND MAINTENANCE.**

We are indeed fortunate to have as our guest editor, Dr. John Metcalf, who is very well known within the REAAA circle.

Dr. Metcalf is the Freeport-McMoRan Professor of Enginering at Louisana State University. He holds the degrees of B.Sc. and Ph.D. in Civil Engineering from Leeds University, England. He is a Fellow of the Geological Society of London, the Institution of Engineers, Australia, and the Institution of Civil Engineers, U.K. and a member of the American Society of Civil Engineers.

Before going to the U.S. in 1992, he was Deputy Director of the Australian Road Research Board. He is the author of some 100 technical papers and has been keynote speaker at several national and international conferences. He has acted as consultant/adviser to UNDP, the World Bank, AIDAB, Kingdom of Saudi Arabia, U.S. DOT, FHWA and various branches of industry.

He is an expert member of the PIARC Committee for Technological Exchange and Development.

Dr. Metcalf's paper **"An Approach to the Use of Non-Standard Materials in Highway Construction"** is printed as the introductory paper for this issue of the Journal.

AN APPROACH TO THE USE OF NON-STANDARD MATERIALS IN HIGHWAY CONSTRUCTION

by

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ABSTRACT

This paper discusses the potential for use of non-standard materials in highway construction. Such materials can be naturally occurring but not traditionally used, municipal refuse, industrial by-products and secondary products to industrial processes. The status and origin of current criteria for construction materials is discussed and it is proposed that many current requirements are conservative and based on empirical studies and experience. Thus there is potential for the use of new materials where the criteria can be developed to more closely reflect the structural properties required in the road pavement. The paper proposes that where new materials do not meet current criteria there is a need to;

* improve the properties of the new materials to meet current criteria,
* challenge the current criteria and improve them wherever possible,
* determine structural behavior of the new materials
* prove the new materials in the field and finally,
* understand them to allow predictions for the future.

INTRODUCTION

This paper gives a personal view of the relevant state of the art in the use of non-standard materials in highway engineering. It discusses aspects of materials and pavement engineering. There is a challenge to make better use of resources, we should never think of a waste product but in terms of secondary or by-product sources of material. Clearly this will require attention to, and resolution of, many problems, technical, economic, environmental and social. Equally clearly, this will not occur without regulation and legislation.

BACKGROUND

A road pavement is an engineering structure required to sustain multiple repetitions of concentrated wheel loads and transfer them to the foundation material for an extended period, while providing an even riding surface. A pavement is composed of one or more layers of materials so assembled as to carry the imposed loads under the prevailing climatic conditions. There are thus four main parameters to be considered. Traffic and climate are essentially given, there is little opportunity for the engineer to change them. The pavement engineer must concentrate effort on providing appropriate materials laid in a layer configuration adequate to withstand climate and load effects.

In more recent times, increasing vehicle speeds, axle loads and traffic volumes have placed new demands for stronger structures with more robust surfacings, and for more stable, durable and fatigue resistant materials. At the same time, resources of traditional materials are becoming depleted and more expensive, and there is pressure to use secondary or by-product materials to alleviate disposal problems.

PAVEMENT MATERIALS

Today's specifications for unbound pavement base course materials have developed gradually since the advent of mechanized construction techniques in the 1930's. However, the demand for smoother, dust free pavements to cater for higher speed, pneumatic tired, heavier motor vehicles, and the need to mechanize construction, led to the use of smaller stone sizes to achieve the surface finish, and to the addition of fine material to "bind" the surface. These factors soon led to the development of the "maximum density" grading where a range of stone sizes is used to pack together into a dense and strong mass.

Densely graded fine crushed rock, being more workable and easier to compact than macadam, largely replaced open-graded pavement materials, is the 'standard' material world-wide.

In the 1930's, a world-wide initiative was successful. This was soil stabilization, in which a stabilizing agent is mixed with an earthen material and the product is laid on the roadbed in the usual way. The agent can be cement, lime, asphalt or a variety of chemicals, or, in the simplest case another soil type. Mixing can be done insitu with specialized mobile mix-in-place machinery, or it can be done in a stationary plant and the mix trucked to the site. On site the material must be rolled (compacted) and finished in the usual way.

The effects of cementitious stabilizers are well established; the more additive the higher the unconfined compressive strength, the better the compaction the higher the strength, the higher the curing temperature and the longer the curing time the higher the strength. In general, the higher the strength the better the stabilized soil will perform as road base, it will have better resistance to the effects of water, longer life and better performance under traffic. There is a trade off, cost increases with additive content and, after a certain point, the behavior of the material layer changes from 'flexible' to 'rigid', that is the material no longer behaves as if it were unbound. This requires a change in the design philosophy for the pavement, yet the mechanisms are not fully understood.

This also means that the true significance of unconfined compressive strength as the criterion for a stabilization process is not fully understood and, therefore, must be called into question. In the interim, means to improve the properties of stabilized materials to meet current criteria must be sought.

These early studies also revealed a feature which continues to be of interest. Little is really known about the behavior of road pavements as engineering structures. We have, since the availability of the computer, ways of theoretically analyzing stresses and strains in multi-layer structures, but the correlations between these analyses and performance in the field are not strong. We are able only crudely to measure pavement behavior in terms of deflections and strains. Our current criteria for pavement materials are based on relatively crude experimentation and on long experience of pavements subject to all the variabilities of materials, construction, traffic and climate. It is little wonder, then, that there is continuing debate about the criteria, and a real need for research into the characterization of pavement materials.

A great deal of research has been done on the properties of granular pavement materials at various moisture and density conditions. Some general rules have been established; for grading, to give maximum density and workability; for particle shape and hardness, to maintain particle interlock; for plasticity and fines content, to bind the aggregate and reduce permeability while maintaining wet strength; for compaction and moisture content, to place materials in the strongest condition. But, the basic criteria are still empirical and in my opinion conservative. They are measures of the intrinsic quality of the granular materials rather than of specific behavior in response to applied load.

Triaxial testing is appropriate to assess shear strength for material selection and pavement design, and a suitable method has been developed in Australia based on the Texas procedure. This modified test is now specified for pavement materials in NSW, without regard to the source of material. That is, no distinction is made between natural gravel, crushed materials, secondary, by-product or manufactured materials, if they pass at the appropriate moisture content.

Many miles of road have been built and still survive without the benefit of sophisticated testing, and with materials which do not meet the current strict requirements for major road pavements. A very wide variety of materials, loams, soft sandstones, natural gravel, decomposed rock and industrial residuals have been and are being successfully used where traffic is not too heavy, the environment is understood and properly taken into account, where design, construction and maintenance is appropriate to the circumstances and quality control is adequate.

There is still debate over the durability of pavement materials, appropriate criteria and mechanisms of degradation. The particles themselves must be intrinsically stable, which means free of degradable clay forming minerals, and hard enough to resist attrition. However, because of our limited understanding of the structural behavior of roads, it is my belief that common current specifications for the selection of base materials have developed to be extremely conservative. It must therefore be possible to use 'non-standard' materials in many circumstances, especially where it is possible to make sure that a worst case condition will exist at such a low probability that a reasonable risk can be taken that the non-standard material will be adequate.

This is particularly true of stabilized materials which have been used for many years and yet for which there are still no fundamental criteria related to the behavior of the material under load in a pavement structure and correlated to the performance of that structure. There is an undoubted need to seek for criteria directly related to material behavior and pavement performance to supplement and replace current empirical specifications.

PAVEMENT DESIGN

Pavement design, in general, is too still an empirical procedure, but recent developments in testing materials and in structural analysis mean that engineering properties can be used in a mechanistic design process to produce more effective pavement designs.

The first use of material structural properties in an attempt to design for load was based on the classic 1885 Boussinesq analysis of stresses in a semi-infinite homogeneous elastic half space. We are now at the beginning of application of computer based techniques which allow for multi-layer 'resilient' structures.

The concept of elastic analysis has developed to a full mechanistic design procedure in the new Guide by which any pavement structure can be designed in a rational and consistent manner. Unusual materials can be accommodated by the suggested characterization methods.

Early attempts to build stabilized bases were not promising because too little pavement stiffness was provided. "Egg-shell" situations were produced where the base cracked extensively, or the blocks resulting from shrinkage cracks were unstable permitting severe pumping where the subgrade was water susceptible. However, longer experience with cracked pavements has shown that, subject to certain conditions, such pavements will continue to behave adequately in the structural sense, though surface appearance may of course be a concern. More recent studies of very heavy duty pavements in the United Kingdom have shown that transverse cracking can originate in the surface of thick asphalt layers and not in the stabilized base or subbase.

These adverse early results slowed the progress of cement stabilization but it is now favored by developments in pavement technology, which allow material stiffness to be quantified and exploited with the mechanistic design approach.

The big advantage of a cemented pavement is its load spreading capacity, because of the relatively high stiffness ratio of pavement to subgrade. The higher this ratio, the greater are the demands on the flexural strength of the pavement structure, the critical property being the tensile strength of the material. Tensile, resilient modulus and flexural fatigue testing is a priority developmental area.

The structural, mechanistic design approach for pavements concentrates attention on the behavior of materials under load, especially repeated load. This requires a shift from classification testing to the determination of strength and stiffness properties for pavement and subgrade materials, under realistic conditions. Repeated load triaxial testing is used for the elastic characterization of unbound materials, and for assessing the plastic deformation properties. Repeated loading or dynamic testing is also needed to determine flexural fatigue and deformation properties of bound materials. As the behavior of stabilized materials appears to show both unbound and bound characteristics, all methods of test must be investigated.

There is a pressing need to develop new test procedures and analysis methods to adequately describe the structural behavior of stabilized layers.

PAVEMENT PERFORMANCE

With the development of structural engineering principles in pavement design and the questioning of empirical test methods and criteria, testing procedures have evolved towards structural rather than physical properties, and towards realistic test conditions. Triaxial, tensile, flexural and fatigue testing in the laboratory are appropriate for materials, but pavement structures are far more difficult to test; even without consideration of cost. Traditional prototype pavement testing, to failure under normal traffic loading, is the most valid form of test, but is too slow for modern development. Thus, some form of accelerated pavement testing is essential to link laboratory characterization of materials to their structural behavior and to field performance of pavements.

A full scale accelerated loading facility (ALF) was designed and built in 1983 in Australia. ALF is a relocatable linear test track, with realistic load simulation and low energy input. Energy is conserved by accelerating and decelerating the load trolley by gravity. Load repetitions are applied rapidly, about every 7 seconds, and the wheel load can be increased to double the legal limit, thus, about 1,000,000 truck axle passes (some 20 year's traffic) can be simulated in 100 days. The nature of the load application has been designed to simulate highway loading in the following ways:

* Uni-directional; indicated as important by the pattern of distress
* Sprung mass; for dynamic loads.
* Driven, rather than towed wheel

The second ALF was built under license in the USA, where it operates at the Turner-Fairbanks research center, the third has been operating in China for over two years, and the fourth has just started working at Louisiana State University.

Australian experience with ALF trials includes;

Somersby, New South Wales: No significant distress was observed under a constant 40 kN load and the trial verified that a single-sized crushed stone macadam sub-base could be used with confidence in heavy traffic applications.

Beerburrum, Queensland: The typical failure mode of a multi layer stabilized pavement was de-bonding of the layers, followed by erosion at the interfaces and the damage in the pavement layers was found to

progress from the upper layers towards the lower layers, contrary to expectations.

Prospect, New South Wales: Over 2 million loading cycles were applied to 18 test sections in order to examine the relative performance of unbound and stabilized blast furnace slag materials. It was found that both the unbound and stabilized slag could be used in place of unbound and stabilized crushed rock provided they were protected from excessive tensile strains by the use of an adequate subgrade and that an adequate wearing course, preferably asphalt, was used to prevent surface wear. The trial confirmed that traditional 'hardness' requirements could be relaxed for softer steel slag.

The use of ALF in Australia has cost some $1 000 000 a year for the past 8 years. The benefits have been estimated at up to $30 000 000 per year. An excellent example of the high rate of return on successful research.

FUNDAMENTAL MECHANISM

The brief review above has concentrated on inconsistencies and inadequacies in current pavement and material design, selection and evaluation practices. There are many reasons for these problems partly linked to the use of very cheap natural materials which exhibit high variability, which are used in structures subject to largely unknown traffic loads and climatic conditions, built in-situ with poorly controlled processes. Less this seem too harsh a criticism of highway engineering, I don't imply any malpractice; it is simply the nature of this industrial process.

However, it is also true that we lack a fundamental understanding of the properties of the materials, the imposed loads and the environment in which they must perform. This is related to the complexity of the problems and to the fact that pavements rarely fail; the only get worse. Thus there has been no imperative to understand fully the fundamental mechanisms.

CONCLUSIONS

The current practices in highway pavement design and materials selection are strongly based on empirical procedures and standards. This makes the prediction of the behavior of non-standard materials and their performance in pavement structures difficult and mitigates against their acceptance and use. The current practices are also conservative, there is room for improvement and cost savings if the current practices can be refined.

There are new testing and characterization procedures for pavement materials and new methods for analyzing the behavior of materials in pavements and, the performance of those pavements in the field. These must now be applied to the new materials.

The opportunity for the application of non-standard materials in practice thus rests on;

* improving non-standard materials to meet current criteria,
* challenging the current criteria and improving them wherever possible,
* determining structural behavior
* proving the new materials in the field and finally,
* understanding them to allow predictions for the future.

To this we must add the concerns with social acceptability, economic feasibility and regulatory acceptance which should form part of a broader program.

EXPERIENCE OF SOIL-CEMENT ROADS IN THAILAND

by

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INTRODUCTION

Soil-cement road was first introduced into Thailand in 1965 by SIAM CEMENT COMPANY(1965) by constructing a test road of 5 kilometers using lateritic soil as a raw material stabilized with cement for serving as base course replacing typical crushed rock which is rare in Northeastern Thailand. Lateritic soil and terrace gravel are local materials abundantly found in Northeastern Thailand, while rock products are deficient. According to Figure 1 it will be seen that rock sources in this region could be found in the southern part bordering Cambodia, and the western part close to Northern Thailand. Road construction in this region has the hauling distance for the rock products of 150 – 300 kilometers. During that time the primary road in this area is the National Highway Route No. 2 which links parts of the Northeast Thailand to Bangkok. Hauling all types of construction materials, especially rock products has to be made along some sections of this road. No other major roads that could be served as convenient routes for hauling the rock products from these sources to the project sites. In order to cut down the construction cost and to lengthen the service life of the roads being used as the hauling routes, lateritic soil-cement seems to be the most economic approach for pavement design and construction during that time. So, after an introduction of the soil-cement test road in 1965 by SIAM CEMENT COMPANY, there was a period of about five years (1967-1971) of using lateritic soil-cement to replace crushed rock for base course construction in Northeastern Thailand.

Some soil-cement roads especially those with asphalt concrete surface, after serving traffic for a while, had a lot of reflected cracks formed on the surface. Routine maintenance method generally applied for repairing the reflected cracks is sealing them with liquefied asphalt or premix. The shown up reflected cracks in soil-cement road made the pavement engineers at that time doubtful about its long-term performance. So it was insisted to stop soil-cement road construction, probably in 1972. During the year of 1967-1971 there were about 1400 kilometers of soil-cement road construction, and some of them are shown in Figure 1.

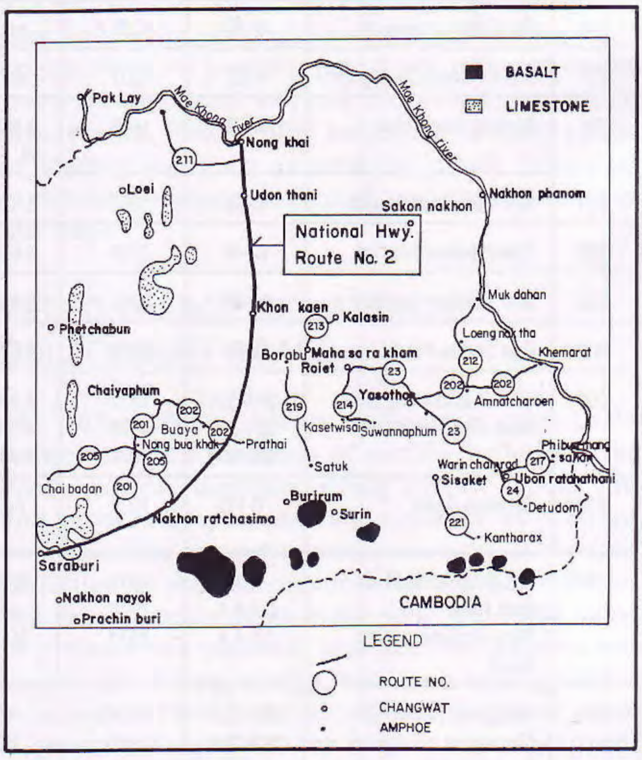

Figure 1: Map Showing Soil-Cement Routes in Northeastern Thailand.

Even though the termination of soil-cement road construction since 1972, research study on long-term performance evaluation of the existing soil-cement roads has been initiated since then.

Table I: Records of Soil-Cement Road Construction and Maintenance in Northeastern Thailand

Route No.	Route	Station Km.	Year Finished	Additives % by wt.	Mixing Process	Surface Types	Constructed by	Records of Periodic Maintenance
23	Mahasarakham-Roi Et-Yasothon-Ubon	120-180 180-212 212-276	1969 1966 1970	4-6.5C,2L 3.4-6C,1-2L 3.4-6C,1-2L	Plant Mix Plant Mix Road Mix	DST[3] DST DST	Thai Japan Sakol Satapat Sakol Satapat	1975[2], 1987 1975 , 1987 1973,1978,1981
24	Varin-Det Udom	4-9	1965	no data	Road Mix	DST	Thai Cement Products Co.	Reconstructed in 1978 and 1980
202	Yasothorn-Amnat Chareon	0-53	1968	no data	Road Mix	DST	Sakol Satapat	1977,1979 Reconst. between Km.33-63 in 1977
202	Amnat Chareon-Khemarath	53-120	1970	no data	Road Mix (Mobile Plant)	AC[4]	CIC	1977,1979,1980
212	Ubon-Mukdahan-That Panom-Nakorn Panom	57-115 115-164 164-212	1969 1970 1970	3-6C,2L 3-6C,2L 3-6C,2L	Plant Mix Plant Mix Plant Mix	DST DST DST	Sakol Satapat Perm Sin Krungtep Sahakai	1977,1980 1973,1977,1981 1973,1977,1981
213	Mahasarakam-Kalasin-Somdet	0-80	1967	3.8-4.5C	Plant Mix	DST	Italian Thai	1974,1977,1979 1980
214	Roi Et-Suwanapoom	0-75	1968	no data	Plant Mix	AC	CIC	1978,1980
217	Varin-Piboon-Chongmek	4-62	1967	no data	Road Mix	AC	Sakol Satapat	1978,1980
221	Sisaket-Kantaralux	0-63	1968	3.5-7.8C 2L	Plant Mix	DST	Thai Japan Construction	1973,1975
201	See Kew-Chaiyapoom	0-64	1967	3.3-4C,2L	Plant Mix	DST	AS	1979,1980,1981
202	Chaiyapoom-Bua Yai	12-48	1969	3-6.2C,2L	Plant Mix	AC	Thanet Porn	1976
202	Bua Yai-Ban Seeda	48-66	1969	3-6.2C,2L	Plant Mix	PM[5]	Thanet Porn	1976
202	Ban Seeda-Patai	66-84	1967	3-6.2C,2L	Plant Mix	DST	Thanet Porn	No seal coat
205	Chai Badan-Nong Bua Koke-Cho Hor(Korat)	244-321 321-340 340-402	1970 1969 1967	3.3-4C,2L 3.3-4C,2L 3.3-4C,2L	Road Mix Road Mix Road Mix	DST DST DST	no data	1979,1981
219	Borabue-Satuk	0-112	1971	no data	Road Mix	DST	Dept.of Hwys. Thailand	1979,1981
2182	Det Udom-Bun Tharix Test Road (Raw material is silty sand)	0-0.6 0.6-1.1 1.1-1.4	1974 1974 1974	2L 3C 5E 3C 3E	Road Mix Road Mix Road Mix	DST DST DST	Dept.of Hwys. Thailand	1980 1980 1980
211	Nongkhai-Tha Bo-Si Chiangmai Test Road (Raw material is lateritic soil)	12.7-12.9 12.9-13.2 9-12.7 31-42	1976 1976 1976 1976	4C 6C 4C 2E 4C 2E	Road Mix Road Mix Road Mix Road Mix	DST DST DST DST	Dept.of Highways, Thailand	1980 1980 1980,1981,1982 1980,1981,1982
2022	Penn-Sumsao Test Road (Raw material is lateritic soil)	1.3-1.5 1.5-1.7 2.5-2.7	1975 1975 1975	3E 3C 2E 3L 2E	Road Mix Road Mix Road Mix	DST DST DST	Dept.of Highways, Thailand	Not seal coat Not seal coat not seal coat

1 Periodic Maintenance means seal coat being applied at particular time
2 Figure indicates the year that seal coat being applied on the surface to seal the crack and induce comfortable riding
3 DST = Double Surface Treatment
4 AC = Asphaltic Concrete
5 PM = Penetration Macadam

RECORDS OF CONSTRUCTION AND MAINTENANCE OF SOIL-CEMENT ROAD

The records of soil-cement road construction and periodic maintenance are shown in Table I. The Department of Highways adopted a criterion of the minimum unconfined compressive strength of soil-cement of 250 lb/in^2 as suggested by ROAD RESEARCH LABORATORY (1958). It is found that 3 to 8 percent cement by weight is adequate to develop the unconfined compressive strength of lateritic soil-cement to 250 lb/in^2 for most lateritic soils with PI less than 18. For those with higher PI, 1-2 percent hydrated lime was added and mixed to the soils to reduce their PI before mixing with cement. For the above range of cement content, unconfined compressive strength of soil-cement mix was recorded to be in a wide range of 110-910 lb/in^2 that shown in Table II, but mostly in a range of 250-520 lb/in^2. In some sections that had the unconfined compressive strength lower than 250 lb/in^2, soil-cement base in these sections would be cored and the cylindrical specimens were tested at some particular period of curing. If the unconfined compressive strength of the cored specimens shows the trends of increasing for about 30 percent over 250 lb/in^2 at the age of about 28 days, that section of the soil-cement base would be considered to be acceptable. If not, reconstruction has to be made. In addition to keeping control the strength of soil-cement, field density of the compacted base must be at least not lower than 95 percent modified Proctor. The average thickness of the compacted soil-cement base will be 15 cm.

Mixing process of lateritic soil-cement might be road mix or plant mix. Both processes could be properly used. However, road mix was found to be more versatile, but the thickness of soil-cement base might not meet the minimum design in some sections. This is probably due to the improper preparation of scarified base before cement spreading and mixing. For the consistent quality of the soil-cement mix and adequate thickness, it is recommended to use the plant mix. Most soil-cement roads were constructed by contractors, and supervised by the Department of Highways. But some particular projects, especially when encountering with some unsolved problems, were constructed by the construction unit of the Department of Highways.

Surface of soil-cement road might be surface treatment, penetration macadam, and asphalt concrete. If the surface is surface treatment or asphalt concrete, reflected crack might be more extensive after the roadway is opened to traffic for a while, especially where the unconfined compressive strength of soil-cement is relatively high. Reflected crack could be cut down where the surface is made with penetration macadam. Conventional routine maintenance to repair the reflected crack is made by sealing them with liquefied asphalt or premix as aforementioned.

Even though it was doubtful about the effectiveness of sealing to seeping water through the cracks, it was found that most soil-cement roads exhibited good performance after serving the traffic for 12-16 years. In other words, all of them have been subjected to 12-16 cycles of rainy season without major failure.

Records of periodic maintenance of soil-cement roads are shown in Table I, and was found that while the roads are opened to traffic for 12-16 years, periodic seal coat was applied for one to three times. Seal coat could be applied on all types of surface. As a result of periodic seal coat as stated, major or plastic failure could be prevented effectively. This result could be justified from the visual evaluation. From the past experience, it could be stated that periodic seal coat on the soil-cement roads tends to lengthen its service life.

Even though plastic failure is not common in soil-cement road, there were two soil-cement roads failed in this pattern after serving traffic for more than 10 years. During the reconstruction phase it was found that the cause of failure is due to the existence of silty clay and water bearing layers underneath the soil-cement base. As a result, quite a few reflected cracks were formed until beyond the scope of routine maintenance. Finally, after subjecting to many cycles of loading and water penetration, plastic failure or even mud pumping was formed, and reconstruction is inevitable.

PERFORMANCE OF LATERITIC SOIL-CEMENT ROADS

The performance of lateritic soil-cement roads could be evaluated on the basis of surface deflection, rut depth, surface roughness, riding quality, service life and records of periodic maintenance as well as stability of the pavement structure as a whole. Surface deflection of the soil-cement road is relatively low, and the surface rut depth is not noticeable. This tends to promote the periodic seal coat to lengthen the service life. Riding quality was found to be good for most routes. The service life of the soil-cement roads is generally longer than the crushed rock base road. On the basis of all parameters evaluated, soil-cement roads in Thailand show a good performance.

RECENT INVESTIGATIONS ON CEMENT TREATED BASE ROAD

All the existing soil-cement roads were constructed following the unconfined compressive strength criterion of 250 lb/in^2. For the actual construction the strength value will be increased up the 520 lb/in^2.

Table II: Summary of Technical Data of Soil-Cement Road in Northeastern Thailand

Route	Station Km.	Year Cons.	Additives % by wt.	Surface Type	UCS[1] lb/in^2	Deflection Test				Roughness Test	
						Year Test	av.X 10^{-3} in	SD 10^{-3} in	av.X+1.5SD 10^{-3} in	Year Tested	RI[2] cm/km.
23	120-180 180-212 212-276	1969 1966 1970	3.4- 6.5C,2L	DST[3] DST DST	250-470 125-600 125-600	1974 1974 1974-76	12-26 16-26 13-30	3-9 4-7 3-8	20-40 27-36 21-41	1978-82 1978-82 1978-82	230-620 180-960 150-1180
24	4-9	1965	no data	DST	250	1975-79	14-23	2-8	18-34	1978	480
202	0-53	1968	no data	DST	no data	1979	10-24	3-7	13-36	1978-82	260-1500
202	53-120	1970	no data	AC[4]	150-more than 250	1975-79	7-17	2-8	11-26	1978-82	60
212	57-115 115-164 164-212	1969 1970 1970	3.2-6C,2L 3.5-6C,2L 3.5-6C,2L	DST DST DST	130-150 110-400 110-400	1975-78 1975-78 1975-79	15-31 11-28 11-30	3-8 2-11 2-8	22-43 14-39 13-44	1978-80 1978-80 1978-80	250-400 100-350 90-260
213	0-80	1967	3.8- 4.5C,1L	DST	160-480	-	-	-	-	-	-
214	0-75	1968	no data	AC	110-more than 250	1974	10-22	3-8	14-35	1978-80	30-330
217	4-62	1967	no data	AC	320-480	1975	9-15	2-5	13-23	1978-80	20-180
221	0-63	1968	3.5- 7.8C,2L	DST	150-450	1975-78	12-33	3-10	14-44	1978-80	80-400
201	0-64	1967	3.3-4C,2L	DST	195-910	1975	14-26	4-8	21-36	1978-80	700-950
202	12-48 48-66 66-84	1969 1969 1967	3-6.2C,2L 3-6.2C,2L 3-6.2C,2L	AC PM[5] DST	250-460 250-460 250-460		no	data		1978-82 1978-82 1978-82	200-420 200-420 200-420
205	244-321 321-340 340-402	1970 1969 1967	3.3-4C,2L 3.3-4C,2L 3.3-4C,2L	DST DST DST	280-460 280-460 280-460	1976 1975 1975	25-42 15-20 21-29	9-13 5-8 8-9	39-57 22-32 33-40	1978-82 1978-82 1978-82	250-480 380-830 240-530
219	0-112	1971	no data	DST	no data	-	-	-	-	1978-82	300-830
2182	0-0.6 0.6-1.1 1.1-1.4	1974 1974 1974	2L 3C 5E 3C 2E	DST DST DST	400-550 30-35 110-140	1974-80 1974-80 1974-80	8-13 11-14 14-19	1-7 2-4 3-5	10-22 15-20 18-27	1977-80 1977-80 1977-80	130-450 100-340 180-210
211	9-12+700 31-42	1976	4C 2E	DST	150-200	1976-80	9-21	1-6	11-27	1977-82	150-1200
	12+700- 12+950	1976	4C	DST	200-280	1976-80	11-16	2-6	14-25	1977-82	300-2200
	13+000- 13+200	1976	6C	DST	300-450	1976-80	11-16	1-4	16-22	1977-82	500-2000
2022	1.3-1.5 1.5-1.7 2.5-2.7	1975 1975 1975	3E 3C 2E 3L 2E	DST DST DST	40-60 150-270 60-90	1976-78 1976-78 1976-78	22-24 25-26 18-23	3-5 4-5 3-4	26-30 32-33 24-29	1977-82 1977-82 1977-82	150-2000 130-1900 100-960

1 UCS = Unconfined Compressive Strength
2 RI = Roughness Index
3 DST = Double Surface Treatment
4 AC = Asphaltic Concrete
5 PM = Penetration Macadam, C = Cement, L = Line, E = Emulsion

Roughness Index is determined by the use of Integrating Irregularity Recorder as manufactured by Tokyo Tanifuji Co., Ltd in Japan

Criteria for the RI value (asphaltic concrete surface) as suggested by the manufacturer are as fallow

0 - 50 (cm/km)	means excellent	50 - 100 (cm/km)	means very good
100 - 200 (cm/km)	means good	over 200 (cm/km)	means bad

As a result reflected crack develops. Recent investigations; both in the Laboratory and in the field, showed that reflected crack could be reduced if the raw soil is well graded. Designing the soil-cement mix to be soft with the unconfined compressive strength in the range of 200-300 lb/in^2 is another approach to cut down the crack formation on the surface. Result of detailed investigations enable the Department of Highways to draft three construction specifications of cement treated materials as follows:

* Specification for Soil-Cement Subbase
* Specification for Soil-Cement Base
* Specification for Cement Modified Crushed Rock Base

Today soil-cement is not only employed in the Northeast Thailand and not only for low volume road as in the past. But it is used for all classes of road. Some major highways having the problem of truck overloading will designed to use soil-cement base or cement modified crushed rock base instead of unbound granular base. Result of pavement analysis by the appropriate computer program shows that cement-treated base pavement is more stable than unbound granular base pavement. It is expected in future cement-treated materials will play an important role in pavement design and construction for the high traffic volume road in Thailand. This is a result of the continuous investigation and research on soil-cement road for about 30 years since the first introduction.

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LOW COST ROADS IN NEW ZEALAND

by

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ABSTRACT

The total length of the road network in New Zealand is over 100,000 km. Historically, New Zealand road authorities have adopted the strategic principle of building an infrastructure based on relatively low cost initial investment in order to provide such a network. Asphalt bound aggregate systems are used for some urban streets and freeways, but 96% of the national highways (11,900 km) are constructed of low cost roads, primarily sophisticated unbound granular pavements with thin bituminous surfacings. Techniques for producing aggregates for unbound granular layers and the seal coat, along with appropriate low cost construction techniques have been developed and are described in this paper.

INTRODUCTION

New Zealand has a population of 3.3 million and an area of 268,675 km^2. The country's road network totals over 100,000 km in length, of which 50,300 km have all-weather surfaces. Maintaining and rehabilitating such a network for relatively low traffic volumes has encouraged the use of alternative pavement designs and practices. Aggregate is generally readily available, though not always of high quality, in most regions of the country. The typical pavement consists of a thin surfacing (most often a sprayed seal coat but sometimes a bituminous mix) directly over unbound granular base and subbase layers.

At present, the maximum gross vehicle weight permitted on national highways is limited to 44 tonnes, and the maximum loads permitted for dual-tyred single, tandem, and triple axle groups are 8.2, 15.0 and 18.0 tonnes, respectively. The New Zealand term for equivalent single axle load is Equivalent Design Axle (EDA). One EDA is defined as the unit of pavement wear caused by one passage of a 80 kN axle load on dual-tyred wheels inflated to 550 kPa. Actual axle loads are related to the reference loads by the *"fourth-power rule"* (the exponent is 4.0). The national highway authority is called Transit New Zealand (TNZ), but prior to 1989, the title was the National Roads Board (NRB).

Flexible highway pavements in New Zealand consist of (Figure 1):

(i) a thin surface which is most often a sprayed bituminous seal with uniform crushed stone chips, but is occasionally a bituminous mix;

(ii) a basecourse which is commonly an unbound granular layer, but is sometimes a lime or cemented layer; and,

(iii) a subbase which is usually one or two (and rarely more) layers of unbound granular aggregate.

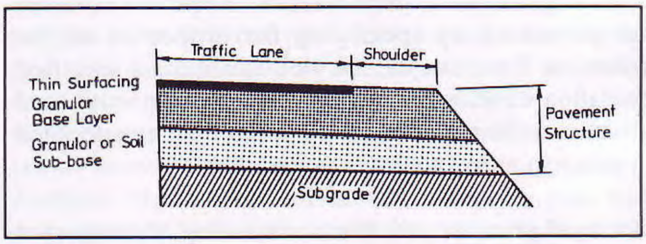

Figure 1: Cross-section of Typical New Zealand Road

Sometimes, one or more layers are stabilised or cemented to improve the properties of poorer aggregates or soils.

The performance model used for thin-surfaced, unbound granular flexible pavements assumes that the surface thicknesses of less than 35 mm do not contribute to the structural capacity of the pavement, and that the stresses are dissipated through the depth of the granular cover layers above the subgrade. The main criteria is the vertical compressive strain in the subgrade because the design theory presupposes that the primary mode of structural failure is permanent deformation in the subgrade. If the vertical compressive strain at the top of the subgrade exceeds the capacity of the soil, then excessive vertical plastic deformation will occur, eventually manifesting itself at the road surface as rutting. The design method is based on a mechanistic or analytical procedure, which uses linear elastic theory to calculate the stresses, strains and deformations in the pavement structure. The calculated response of the pavement is based on

the subgrade strength, thickness of the different pavement layers and the compaction level of each layer. The flexible pavement thickness design charts in the State Highway Design and Rehabilitation Manual (12) are derived from the subgrade strain criteria for flexible pavements in the Shell Pavement Design Manual (11), with some adjustments based on research and local empirical data. The subgrade strain criterion for primary highways in New Zealand is:

$$\epsilon_{cvs} = 0.021\ N^{-0.23} \quad (1)$$

where

ϵ_{cvs} is the vertical compressive strain in the subgrade, and

N is the number of repeated equivalent design axle loads.

For comparison, the Australian subgrade strain criterion is:

$$\epsilon_{cvs} = 0.0085\ N^{-0.14} \quad (2)$$

Even though the design of pavements consisting of unbound materials is based primarily on one criteria (to limit the vertical compressive strain at the top of the subgrade), other possible modes of primary failure (such as a shear failure in the basecourse) are prevented by specifying the properties of the materials. For example, the basecourse has a specified gradation envelope to ensure proper compaction and effective drainage, and other properties are specified in order to avoid material degradation.

Unbound granular pavements consisting of compacted well-graded crushed aggregate can sustain large numbers (over 1.5×10^6) of 80 kN axle load repetitions in the absence of deleterious ground moisture and environmental factors (9). Trials involving instrumented pavements in service and in the Canterbury Accelerated Pavement Testing Facility have shown that actual vertical compressive strains in the top of the subgrade are up to three times the theoretical maximum allowable, depending on the criterion used, which suggests that the criterion on which the pavement thickness design charts are based could be conservative. However, the strains predicted by the Australian model (Eqn. 2) are closer to the measured strains.

PROPERTIES THAT DETERMINE AGGREGATE QUALITY

In New Zealand, high quality aggregates of crushed rock are produced to provide sufficient:

* Stability to satisfy the service requirements and demands of traffic without distortion or displacement;
* Voids in the compacted aggregate to ensure adequate permeability and to allow for slight additional compaction under traffic loading;
* Workability to permit an efficient construction operation in the placement of the aggregate; and,
* Strength and durability for a long life without need for repairs.

The cost of constructing roads using high quality crushed aggregates is substantially less than the cost of asphalt bound and rigid pavements of equivalent strength. Aggregate strength and durability depend on a number of factors, including particle size distribution or gradation. Well graded aggregates have just enough particles of a given size available to fill the voids between the particles of the next larger size, which leads to good uniformity and smoothness of the finished surface. Coarser gradations are cheaper to produce because they need less crushing and have less surface area, but excess coarse aggregate leads to segregation problems if used in the base course. Excess medium-sized aggregate leads to open gradations which increases perviousness, and excess fines lead to weakening of the base support with increases in moisture content. The gradation envelopes for premium unbound aggregates in New Zealand create a well-graded aggregate with just adequate voids for permeability, but coarse graded aggregates are used in the subbase layers.

The particle size distribution is determined from Talbot's equation:

$$P_d = 100\left(\frac{d}{D}\right)^n \quad (3)$$

where

P_d is the percentage smaller than particle size d,
d is any particle size less than D,
D is the nominal maximum particle size, and
n is an index of porosity.

Higher 'n' values produce higher void contents (assuming particle shape and other parameters are constant). For example, n of 0.4 and 0.65 produce a fine (or dense) grading and coarse grading, respectively. Aggregate satisfying the New Zealand standard lies within the envelope created from the limits of n = 0.40 and n = 0.6. The envelope of gradations is shown in Table I (14). The shape of the gradation curve is also controlled to limit local variations in the grading envelope; Table II specifies the proportion by weight of material in each fraction. However, these envelopes are difficult to achieve, and the aggregates do not always satisfy the standard.

Aggregates with a larger maximum size give higher shear and compressive strengths, but the maximum

Table I: Gradation Fraction Limits

Fractions (mm)	Percentage of Material within the Fraction	
	AP40	AP20
19 – 4.75	28 – 48	–
9.5 – 2.36	14 – 34	20 – 46
4.75 – 1.18	7 – 27	9 – 34
2.36 – 0.600	6 – 22	6 – 26
1.18 – 0.300	5 – 19	3 – 21
0.300 – 0.075	2 – 14	2 – 17

Table II: Gradation Envelopes

Test Sieve Aperture	Percentage Passing	
	AP40	AP20
37.5 mm	100	–
19.0 mm	66 – 81	100
9.5 mm	43 – 57	55 – 75
4.75 mm	28 – 43	33 – 55
2.36 mm	19 – 33	22 – 42
1.18 mm	12 – 25	14 – 31
600 μm	7 – 19	8 – 23
300 μm	3 – 14	5 – 16
150 μm	0 – 10	0 – 12
75 μm	0 – 7	0 – 8

size in basecourse aggregates is limited to 40 mm to enhance workability, to minimise segregation potential and to allow for the optimum thickness of the layer being placed. In surface courses, the top size is usually limited to 20 mm to minimise problems with workability.

Fine particles affect most aggregates detrimentally, but some fines are necessary for cohesion and proper gradation. The optimum fines content is about 5%; the New Zealand specification for unbound aggregates specifies a maximum fines content of 8%. Too high a fines content inhibits permeability. The purpose of the Sand Equivalent test is to provide a measure of the proportion of the fine material in an aggregate containing coarse particles. Clay particles in aggregates can substantially reduce permeability and increase susceptibility to pore pressures, which decreases stability. The test uses all the material passing the 4.75 mm sieve. The sand equivalent or SE is a ratio of the volume of sand to the volume of sand plus fines. The advantage of this test is that it is sensitive to both activity and relative proportions of fines. The SE test is used instead of the Plasticity Index test, but results of the two tests are not comparable. The presence of fines is limited by specifying that the SE cannot be less than 40 (6).

Durability is the resistance of an aggregate to degradation and disintegration. Degradation refers to surface wear and internal fracture and depends on: rock type; gradation (openness increases degradation); particle shape; particle size (larger particles degrade more); compactive effort; and other factors. Degradation is minimised by using dense gradations, limiting the maximum size of particle, crushing aggregate to create cubicle shapes, and applying heavy compactive effort to force breakdown during construction. The crushing resistance of the material must be 130 kN or greater to provide adequate strength (5). Disintegration refers to chemical weathering and leads to the formation of fines. The weathering resistance test is an accelerated laboratory test used to assess the resistance of aggregate to wetting and drying, and heating and cooling cycles.

Particle shape influences compaction and performance. Rounded particles compact more readily than angular particles but the former push or shove under the roller. Cubical particles will fit together tighter than either round or splintery particles and will resist shoving and degradation. Therefore, New Zealand highway specifications require that the proportion of broken rock, by weight, in each of the three fractions of aggregate between the 37.5 mm and 4.75 mm sieves must be greater than or equal to 70%, to ensure that the aggregate contains sufficiently sharp surfaces to enhance inter-particle friction, but this requirement was not always satisfied.

Surface texture is also important in ensuring good quality unbound basecourse aggregate. Crushing aggregates increases surface roughness as well as angularity. In a compacted mass, angular aggregates will have abutting faces rather than points of contact, thus surface texture influences resistance to displacement between particles.

Until recently, the road aggregates in New Zealand were created by crushing alluvial gravels or rock, then screening and sieving the crushed material. However, the method did not guarantee the correct gradation was always achieved, nor did it ensure that the desired percentage of angular faces was provided. Now, a new patented process involving three crushing stages feeding source material to a compact, portable blending plant (called BlendTech) is available commercially. The process can produce any specific gradation and ensures that the required angularity is achieved at the construction site or in a quarry, for

relatively low cost. The portable blending plant separates discrete fractions of the crushed material into bins, then recombines the fractions into the required gradation; any excess material is fed back into the processing, thereby enhancing economic efficiency. The road aggregate produced from this process exceeds the current standards needed for construction.

CONSTRUCTING UNBOUND GRANULAR PAVEMENTS

The objectives when constructing unbound granular pavements to a high standard are:

* design thicknesses must be achieved consistently within small tolerances;
* the material must be compacted uniformly; and,
* the surface shape provide adequate drainage, safety and comfort for the road user.

The loose layer thickness must be uniform to ensure that full depth compaction is achieved in the pavement course and that a sufficient thickness is provided for placing and compacting. The placing of aggregate must be controlled and working minimised to avoid segregation. The aggregate should always be placed directly from the spreading equipment in the correct loose layer thickness so that, if there is a need for a grader, it is only to trim the high spots. End dumping of loads of aggregate and then spreading is unsuitable because the working of the material leads to segregation. Paver, transverse bottom dump, roll spreader or controlled tailgate spreading are satisfactory, depending on operator expertise. Layer thicknesses are calculated and controlled, to prevent uneven surfaces after compaction. However, minor excess thicknesses can be cut off or an additional thin layer of aggregate can be applied during vibratory compaction, before the surface is tightly compacted.

The two principal objectives in compaction of pavement layers are that the compaction is uniform to the underside of the layer and that over-densification, usually by degradation or breakdown of the aggregate, is prevented, especially in the upper 30-50 mm of the basecourse layer which is to be sealed. Non-uniform compaction of a basecourse causes early wheel track rutting in service. Over-densification can result in shallow shear under early trafficking. The vibrating roller, by its compaction process, prevents the non-uniform compaction with depth, whereas both vibrating and non-vibrating steel rollers can produce over-densification if excessive rolling is applied. Therefore, static weight rollers are precluded for all construction except for the finishing of the surface.

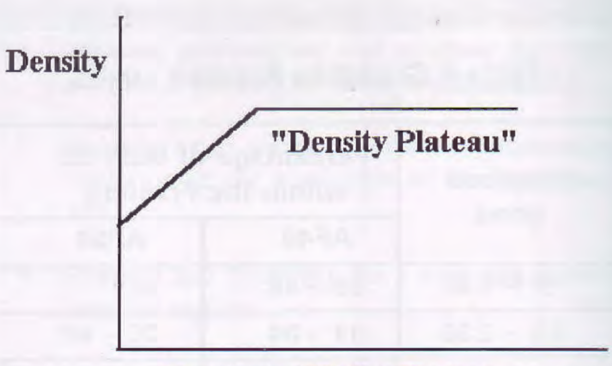

Figure 2: Compaction and Density Plateau Concept

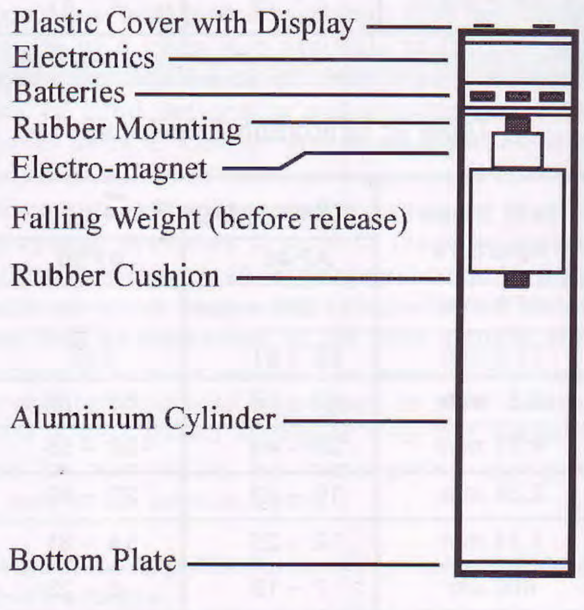

Figure 3: Portable Falling Weight Deflectometer (Loadman)

New Zealand engineers rejected the practice of compacting to a laboratory-derived density over ten years ago because of the inherent deficiencies in such a method. Primary compaction is completed using the minimum necessary number of passes of a vibratory roller of specified characteristics needed to achieve an uniform dense condition. The density or modulus increases substantially until a plateau is reached, then changes slightly or not at all (Figure 2). The structural capacity of the granular layer being compacted is measured after each pass of a roller, and once two successive readings show no change or a slight decrease in density, compaction ceases. Subsequent compaction by normal wheeled traffic consolidates the surface.

Compaction in the field depends on so many factors, many of which cannot be replicated in laboratory determinations of densities of samples. Thus, compaction is monitored by measuring densities using a nuclear density meter in the back-scatter mode on the same spot after each pass of the roller. Another

device, the Clegg Hammer Impact test, has been widely used in the past few years to indicate surface uniformity. Recently, a more accurate device, the Loadman, has become commercially available; the Loadman is easy to operate and to transport. The Loadman consists of a cylinder which contains the electronics and a mass (Figure 3). The mass is released from an electromagnet and drops a constant height onto a base loading plate; different diameters of base plate can be attached. The output from an accelerometer is used to determine the vertical displacement of the pavement layer. Using the displacement, the dropped load and the area of the base plate, electronics within the Loadman calculate the elastic modulus of each layer, and the values are displayed instantly. The density meter or the Loadman can be used to check that the maximum strength has been achieved uniformly over the entire compacted area. By not relying on a laboratory-derived value, the method allows for the normal variation in the great volumes of unbound road aggregate used in New Zealand.

By the above method, over-compaction (which can cause aggregate degradation, reduction in density at the surface of the layer, and segregation) is prevented also, while still gaining the maximum possible compaction. This also prevents 'bounce' of the vibratory roller drums. Bounce of the vibrating roller indicates excessive compaction and cannot be allowed; no further vibratory compaction should be applied to an area in which bounce has occurred.

On completion of the compaction of a basecourse, the surface is often too pervious and thus unsuitable for sealing; sometimes it may even be insufficiently stable for uncontrolled traffic use. Therefore, a "running course" (maximum particle size of 20 mm) is spread as the upper most layer of unbound aggregate. The main purpose of a running course is to provide a lubricant effect between traffic tyres and the compacted surface, not unlike that of a layer of marbles, to protect the constructed pavement from excessive traction, braking and turning effects of road tyres. Under the kneading effects of rubber tyred rolling and controlled road traffic, the running course will abrade the surface and loosen excess fines. The finer particles densify and stabilise the surface texture, which is beneficial for the seal coat and for resisting traffic abrasion (13).

The aim is to achieve a layer one particle thick over the entire pavement surface regardless of the size of particle. A uniform spread of running course aggregate, particularly in the wheel tracks, is maintained by dragbrooming. Usually, at least 1000 vehicles are needed over the unsealed surface to complete the process. Ten passes of supplementary rolling by a pneumatic-tyred roller (minimum weight of 7 tonnes spread over at least seven pneumatic tyred wheels) is specified for road sections having AADT of less than 600 vehicles per day. The normal road traffic is channelised by defined lanes, and, combined with frequent transverse shifts of those defined lanes, ensures an uniform surface and prevents the development of rutting during the initial trafficking period, while the final surface stability is developing.

DESIGNING SEAL COATS FOR LONG-TERM PERFORMANCE AND HEAVY AXLE LOADS

Because of the relatively high cost of bitumen in New Zealand, seal coats are the most attractive economic alternative for providing all-weather roads. Common types of seal coats are illustrated in Figure 4. The functions of the seal coat are to provide an impermeable membrane over the basecourse and a skid resistant surface, as well as a wearing surface. The design of New Zealand seal coats is based on the theory and mechanisms proposed by Hanson (2), who related the bitumen application rate to the size of the stone chip, the ratio of the chip's average, least and greatest dimensions, and the residual void space

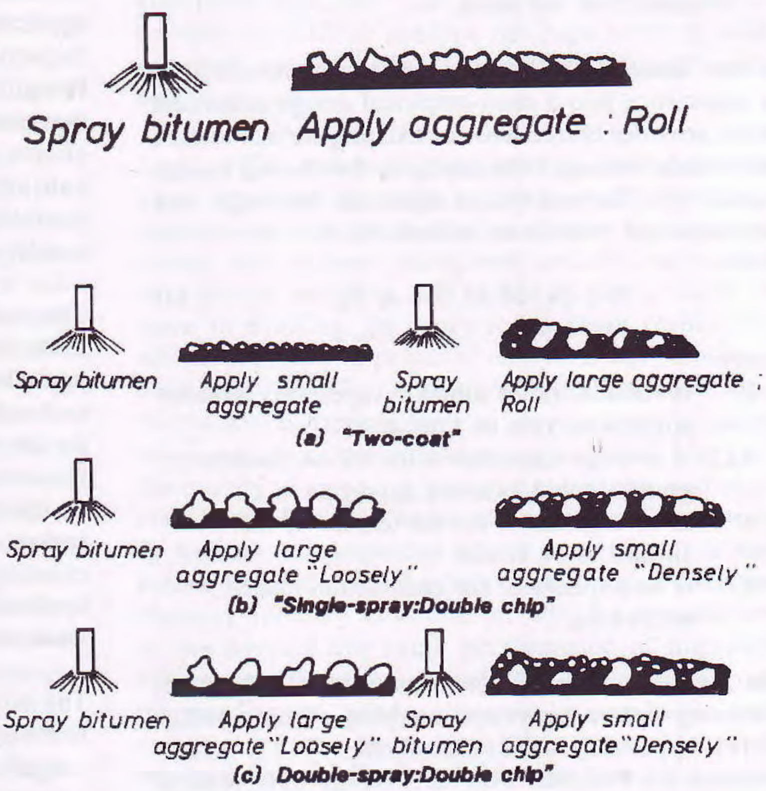

Figure 4: Chip Seal Systems and Terminology

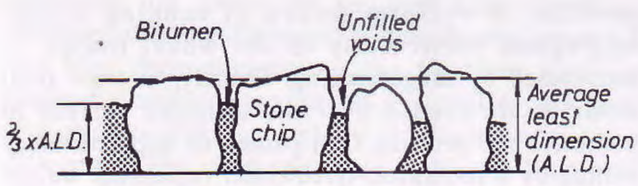

Figure 5: Cross-section of a Single Coat Seal

within the single layer thickness of the aggregate cover. Later, the major assumptions were refined (4):

1. When one-size cover aggregate is spread over a bitumen film, the particles lie in random positions and the voids between the particles are approximately 50 %.

2. Rolling partially reorients the aggregate particles and reduces the voids to about 30 %.

3. Finally, after considerable traffic, the particles become oriented into the densest matrix, all lying on their flattest sides, and the voids are reduced to approximately 20 %.

4. Since the particles lie on their flattest sides, the average thickness of a surface treatment is the average least dimension (ALD) of the stone chips, as shown in Figure 5.

5. For good performance, under the typical traffic volume of 500-1000 vehicles per day, the quantity of asphalt binder used should fill about 70 percent of the voids.

In New Zealand, the basic precepts have been refined by experience into a semi-empirical design procedure which provides corrections for existing surface texture and vehicle loading, culminating in the *Sealing Design Manual* (1). The seal design algorithm for single coat first seals and re-seals of cutback bitumen is:

$$R = (0.138\ ALD + e)\ T_f \qquad (4)$$

where

- R is residual (after diluent evaporates) bitumen application rate in l/m^2 at 15°C
- ALD is average least dimension of the uniformly graded, cubic, crushed aggregate
- e is an adjustment for the texture of the surface to be coated
- T_f is an adjustment for compaction due to trafficking.

The algorithm is based on observations and studies involving public highways carrying normal traffic, which typically consists of 10 to 15% heavy commercial vehicles with an average axle load of about 5 tonnes. The surface texture is quantified by the sand circle test. Additional design algorithms have been developed for two-coat seals and emulsified bitumens.

FACTORS AFFECTING SEAL COAT DESIGN AND PERFORMANCE

80/100 penetration grade bitumen is preferred in the warmer regions north of central North Island, and 180/200 grade bitumen is used throughout the remainder of the country. A rational basis for both modifying the bitumen with diesel and for temporarily softening it with kerosene was introduced in 1965 (3). Laboratory trials established the upper viscosity at which the various types of stone chip could still firmly adhere to a freshly sprayed bitumen film. The road surface temperature was assumed to be a function of the ambient air temperature and the percentage of cutback (usually kerosene) was adjusted accordingly to produce a target viscosity at the time of spraying.

Subsequently, information on the viscosity-temperature-cutback relationships for bitumens used in New Zealand has been extended, and modern instruments have enhanced measurement of the true surface temperature. Field measurements have shown that the relationship between air and road temperature is rather more complex (8). Nevertheless, the basic principles have remained the same.

In addition to material properties and environmental factors, seal coats are very dependent on operator skills and equipment precision. Fortunately, under normal traffic loadings, *"errors"* in bitumen application arising from incorrect design assumptions, departures from theoretical binder formulation, irregularities in sprayer performance or minor departures from specified practices tend to negate the effects of each other. Moreover, a typical seal coat subjected to common loading conditions has considerable inherent tolerance. Adhesion agents are usually added to the bitumen.

The main causes of poor chip seal construction used to be incorrect bar heights and worn, misaligned slot jets (slot jets predominate in New Zealand, for both cutback and emulsion spraying). *A Specification for Performance of Bitumen Distributors* (14) was introduced to ensure an application rate precision in the order of +/-2.5 %. Subsequent testing of sprayers indicated that some did not have the precision essential for producing uniform bitumen films at low application rates, principally because the sprayers had not been designed for such duties.

The cover aggregate used in New Zealand seal coats is always crushed stone particles of uniform size and a cubic shape, even though this is more expensive than a graded cover aggregate. Particle size range and shape has been tightly specified and controlled for many years, so that a good mosaic is produced in the seal

coat cover aggregate. The design procedure assumes that the void volume is still twenty percent, though modern stone crushing plants produce a more cubic chip than the norm of fifty years ago.

Until recently, it was also believed that heavy rollers were essential to chip embedment but this apparently self-evident premise has been disproved (7). The research indicates that the mass of the roller compactor is less important in creating a tightly locked mosaic of the stone chips than tyre action. The research also showed that excess cover aggregate interferes with particle placement and early alignment under trafficking, both of which are essential for proper embedment at low bitumen contents. Roque *et al* (10) found that no more than one pass of an 8 tonne pneumatic roller was needed to compact cover aggregate.

In spite of the theoretically rigid requirements, it is not uncommon practice in New Zealand for contractors on private work to exercise an appreciable degree of experience-based judgment in determining the appropriate bitumen and aggregate application rates for specific situations. The application rates of bitumen tend to be higher to avoid risking loss of stone chip. As a precaution against loss of chips by traffic action the actual application rates of cover aggregate also tend to be higher than the rates derived from theoretical design procedures.

In recent years, increasing lengths of private, sealed roads have been built by logging companies in the forest plantations of New Zealand. Normal state highway chip seal design and construction standards are being used on arterial forestry roads, but the axle loads are not subject to the load limits imposed on the national highway systems, so some of the unbound granular pavements are subjected to up to 16 tonnes per axle in multi-axle groups, which is twice the current public highway limits. Chip seals are an attractive economic means of providing an all-weather surface for the loadings that are being experienced on the arterial forestry roads. The requirements of low volume arterial forestry roads subjected to heavy axle loads are actually superior to those acceptable for state highways, and chip seals designed using existing procedures experience severe bleeding under the heavy axle loads. Experimental work has been instigated to further the development of seal coat design, construction techniques, and specialised materials suited to the special requirements of forestry roads sustaining heavy loads. Results of fifty field trials implemented thus far have shown that enhanced quality control during construction, reduced application rates of bitumen, and use of polymer-modified bitumens are providing the required performance. Sealing practices, especially with respect to bitumen and stone chip application rates, additions of cutback diluent, and adhesion agent, requires supervision and equipment able to readily comply with the more rigorous requirements.

OBSERVATIONS

The roads constructed using the above low cost methods are performing well, but many roads are being resurfaced currently. New Zealand road authorities are spending over $NZ 600 million annually on maintenance and resurfacing, but relatively minor amounts on major rehabilitation or new construction. The low cost roads described above also do not necessarily provide the lowest user costs. Chip seals containing quality stone can provide excellent skid resistance, but also increase rolling resistance and noise. The low cost roads tend to become rougher sooner than some more expensive alternatives, and this leads to increased vehicle wear and fuel consumption. The more frequent maintenance required by low cost roads increases the user delay costs, and the application of excess cover aggregate increases the likelihood of vehicle damage due to loose stone chips. On the other hand, the relatively high proportion of the total New Zealand road network (over 50%) having all-weather surfaces was feasible only through the development of the expertise and the procedures described in this paper.

Because of the difficulty in constructing long-lasting unbound granular pavements and durable chip seals to support reasonably high traffic volumes and medium axle loads, method or 'recipe' specifications have been developed and applied to road construction. However, the roading industry has evolved and matured, and the trend now is to introduce end-result or performance-based specifications, but this requires substantial re-training of staff to achieve the high level of skill required for the successful introduction of such specifications.

Since 1990, a competitive pricing procedure has been applied to state highway construction, maintenance and rehabilitation, which is supposed to take into account intangibles and life cycle costs of highway projects. All public highway work is open to bidding. In order to increase respective market shares, unit prices of materials and resources have been drastically reduced. At the same time, contractors and consultants in the pavement industry have implement quality assurance schemes. Much of the quality of unbound granular pavements and chip seals depends on the skills of experienced operators as well as the properties of the materials. It is the author's opinion that the long-term effect of the new highway industry environment will be a reduction in the overall life cycle performance of highway pavements. However, it is in the best interests of the road users, road authorities, consultants and contractors to ensure an efficient, viable road network, so once the industry settles down to a more normal, responsible attitude, the quality of road work should improve once again.

CONCLUSION

Adequate pavement performance under moderate to heavy traffic volumes can be achieved using thin-surfaced, unbound granular pavements. The main criteria applied in the thickness design of such pavements is to limit the vertical compressive strain in the top of the subgrade to a sustainable magnitude. The basic principles for constructing unbound granular pavements are uniformity in the aggregate, the pavement layer thickness, compaction, finished surface texture and surface level, and minimal working of the aggregate by placement and compaction equipment. Thus, a rigourous standard is applied to create high quality aggregates of crushed rock or gravel, which increases their cost but reduces the overall cost of road construction. Also, compaction techniques have been developed to achieve the optimum conditions in the pavement layers for the lowest cost.

Properly placed and compacted seal coats consisting of cubic, crushed chips of uniform size and the correct bitumen quality and grade provide excellent, long-life all-weather surfaces, even under heavy axle loads. However, even though unbound granular pavements surfaced with chip seals do not necessarily provide the lowest overall cost to the road users, such pavements were needed to maximise the total length of the all-weather road network for the relatively low traffic volumes and sparsely populated regions of New Zealand, and thereby enhanced the economic development of the nation.

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FIT FOR PURPOSE RURAL ROADS

by

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INTRODUCTION

Whilst not overlooking the strategic importance of our rural highways, rural primary and secondary roads are the backbone of regional development. Apart from delivering community service obligations and fulfilling social and communication needs, these roads, often long with relatively low traffic volumes, service remote communities providing the catalyst of successful primary industry developments.

As engineers and managers, road infrastructure development is a relatively simple matter if we adhere to the use of proven methodology and techniques using conventional and often expensive construction materials to develop these rural roads.

THE NORTHERN TERRITORY

The Northern Territory has spurned this option in favour of achieving low cost, fit for purpose designs which utilise locally available natural materials and suit its tropical and arid climates and sparse population. The Northern Territory consists of a vast land area (1,346,000 km}) with a road network of some 22,000 kms ranging from flat bladed track through formed and gravelled roads to sealed national highways and high quality duplicated urban arterials.

All this is encompassed by a Territory of diverse geographic and climatic variation; from the monsoonal sub-tropics of the north with an annual rainfall in excess of 1,600 mm (all of which falls between November and April) to the dry arid central region with an annual rainfall of just some 200 mm.

The Northern Territory has a long history of using natural gravel as its primary roadmaking material.

The first substantial road development to sealed standard was undertaken during the years of World War II, particularly 1942–1945 when the Stuart Highway was sealed between Alice Springs and Darwin. At the same time, a number of airstrips were also constructed from natural gravel and sealed.

During the latter part of the 1960's, a substantial development program of providing 'beef roads' – roads to serve the expanding cattle industry – was undertaken. This involved the development of all weather standard roads sealed to a width of 4 metres.

Post Self Government, July 1978, a further surge of road development was evident. This surge was in part to meet desires frustrated under the previous Federal administration and to meet community service obligations and growing development needs.

Small isolated communities and isolated rural cattle properties mean that many rural roads have small traffic volumes, some less than 100 v.p.d. .

In order to meet ever increasing community expectations with regard to road quality and availability; the Northern Territory Department of Transport and Works has been forced to become progressively more innovative in its approach to providing road infrastructure.

The basics of road construction have changed very little over the past twelve to fifteen years. Geometric design standards, the use of natural gravels for pavement – both sealed and unsealed, the use of sprayed chip seal for surfacing. What has changed in the design philosophy applied to the engineering of water across roads and the use of what previously would have been called more marginal materials.

MARGINAL MATERIALS

The use of lateric gravels for most roadwork in the Northern Territory is based on significant service history. These materials are abundant and can generally be won from close proximity to the road being constructed.

Table I gives the current specification gradings for both sealed and unsealed roads whilst Table II describes the associated properties of each particular product.

AUST STANDARD Sieve (mm)	Percentage Passing by Dry Weight			
	1	2	3	4
75.0	100	–	–	100
/37.5	80-100	100	–	80-100
19.0	50-80	70-100	100	60-100
9.5	35-65	50-80	70-100	50-95
4.75	25-50	35-65	50-80	40-80
2.36	15-40	25-50	35-65	30-65
0.425	7-20	10-30	15-35	20-50
0.075	3-13	4-16	6-20	5-25

Table I: Current Specification Gradings for Both Sealed and Unsealed Roads

	Sealed Base	Shoulder Material For Sealed Base	Unsealed Base/ Sub-Base
Liquid Limit (LL)	25% max.	35% max.	30% max.
Plasticity Index (PI) Linear Shrinkage (LS) PI x % passing 0.425 mm sieve	1 – 6 0 – 3 180 max.	4 – 12 2 – 8 400 max.	1 – 10 1 – 5 400 max.
California Bearing Ratio (CBR) 4 day soaked at compaction	 80 min. 100 %	 50 min. 95%	 30 min. 95%

Table II: Associated Properties of Each Particular Product

Work Components	Mean Dry Density Ratio (R)%
Natural Surface to 150mm below Subgrade Surface. Fill	90.0 or greater
150mm below Subgrade Surface, Subbase Shoulders, Select Fill	95.0 or greater
Base Course	100.0 or greater
Stabilised Base Course	98.0 or greater

Table 3: Compaction Standards (to Australian Standard values)

In practice, it is not unusual to accept materials which have non-conforming gradings and are out of specification on Plasticity Indices. However, the CBR of the material is never reduced.

In the arid region of Central Australia, where laterites are not as abundant as in the rest of the Northern Territory, sand clays and calcretes are commonly used as pavement materials and perform admirably in this environment.

SURFACE SEALING

Sprayed chip seals have always been the standard surfacing for roads in the Northern Territory. Asphaltic Concrete has been used in urban areas, on special applications and some industrial zones, and slurry seals are gaining a wider application as an alternative to asphaltic concrete for some applications.

Chip seals are widely used throughout Australia, but the variations possible, particularly in remote locations, see them at their versatile best. Simplistically, Class 170 or 320 cutback binder is sprayed hot and immediately covered with an aggregate surface dressing. The selection of aggregate provides the practitioner with significant choice in terms of cost and durability.

The standard practice is to use a crushed single size aggregate in the 7mm – 14mm range. Alternatively, screened, crushed or uncrushed river gravels have proved very durable and can be very economical if won close to the site of the works. Other aggregate alternatives which have been used very successfully are screened Laterite and coarse sand. The coarse sand is the least durable but can still be cost effective in very low traffic remote, locations where sealing might not otherwise be viable.

DRAINAGE PHILOSOPHY

Provision of floodways has long been an integral part of road design philosophy. Given the nature of terrain and monsoonal downpours experienced in the top end of the Northern Territory in particular, it is often not economically feasible to put all floodwaters under the road. Until the late 1980s, flood immunity associated with design storm events saw all drainage structures and provisions decided on a trafficable flood event, even if overtopping occurred.

The introduction of the concept of Annual Average Time of Submergence (AATOS) in determining immunity levels has seen very significant cost savings, particularly in flood plain areas. The principal savings are effected in lower gradelines and hence earthworks volume reductions. Smaller size culverts and floodways also predominate.

The use of floodways is dominant in Northern Territory road design and is the most cost effective method of managing flood waters in our climate.

BITUMEN EMULSION STABILISATION

Unsurfaced pavements or shoulders are of course always exposed to degradation by traffic and wind and water. We have begun to experiment with the application of low concentration of bituminous emulsions to stabilise non-plastic Fine Crushed Rock and lateric gravels.

The materials to be stabilised are usually lightly typed and remixed with water containing 10% bitumen emulsion (60% bitumen). This gives a residual bitumen content of around 0.36%.

At about half the cost of sealing, the benefits are significant:

* uses 'low tech' equipment – water cart, grader, roller;
* does not have the recurrent costs of sealing;
* is readily reworked in the event of pot holing or other forms of failure; and
* decrease in permeability and water sensitivity.

EXPANSIVE CLAY SUBGRADES

Many of the flood plains in the northern part of the Northern Territory consist of 'black soil' or extremely expansive clays. This material is often used for the purposes of forming the road prior to pavement construction and sealing.

The wetting and drying cycle which is part of our monsoonal climate can result in significant pavement heave and cracking. Conventional rehabilitation such as recompaction and/or gravel resheeting produce only short term benefits; to replace the expansive clays is prohibitively expensive.

It has been reasoned that movement of the embankment and pavement can be reduced by controlling the moisture content of the material below the pavement.

A plastic membrane is layed over the entire formation from table drain to table drain see Figure 1. Principal criteria for the membrane is that it should be durable and able to withstand loads imposed by construction machinery.

Table 4 gives the technical properties of readily available membrane material used.

Body Dart Impact Strength (ASTM D1709-62T)	550g
Tear Resistance (ASTM D1922-67) Machine Direction Transverse Direction Puncture Resistance Thickness	 19.24N 24.64N 75N 300 microns

TABLE 4

It has been found that by pre-mixing the pavement material and placing with a scraper, minimal damage is done to the membrane. Pavement is then graded and compacted in the conventional way. Careful attention to batter slopes and location of table drains ensures that drainage is kept at the greatest possible separation from the road pavement.

The resultant stabilised moisture in the road embankment appears to be effective when compared with control sections established adjacent to the repaired area.

Whilst a true benefit of this treatment can only be established over a long period and initial costs are somewhat higher than conventional rehabilitation, the recurrent costs of the conventional treatment far outweigh the predicted costs of this technique.

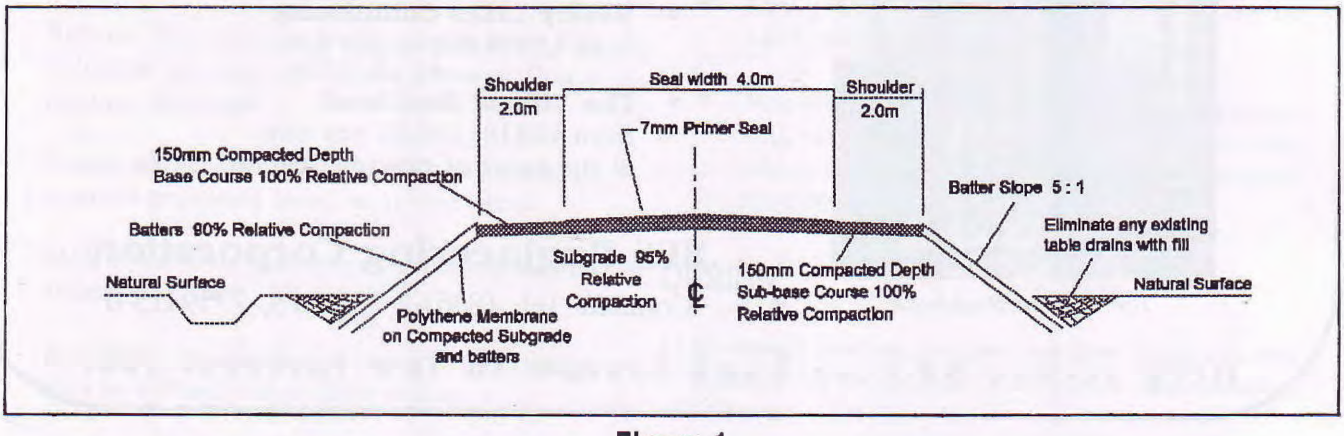

Figure 1

CONCLUSION

Whilst the technique and ideas contained in this article are limited in number and technical detail, they are but some of the innovations and trials conducted by the Department of Transport and Works.

The Department does not have a formal Research program or a Research and Development Department. However, far from discouraging innovation, it may in some ways promote it. Individuals are encouraged to innovate over the full range of our functions – from guide posts to traffic signals; warning signs to controlled water flow.

ACKNOWLEDGMENT

This article is provided with the permission of Mr C J Fuller, Secretary to the Department of Transport & Works.

The views expressed are those of the author and not necessarily those of the Department of Transport & Works.

THE OPTIMUM OF LOW COST LOW VOLUME ROAD

by

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INTRODUCTION

Over the last few decades, discussion on Low Cost Low Volume Road has been of interest to relevant engineers in many countries. This is not surprising because low volume roads will always be needed by all countries in the world to facilitate their "under-developed areas, for example, rural areas, scarcely-populated areas, and remote areas. In general, provision of low volume roads demands low standard specifications as well as low costs.

The Low Cost Low Volume Road has been discussed by road engineers in many meetings, for example, local, regional and international seminars, workshops, and conferences. Recommendations on planning, design, construction and material requirements of Low Cost Low Volume Road have resulted from the discussions. This enables engineers to carry out their task of providing a better road.

The paper discusses important aspects to be taken into account in providing an optimum Low Cost Low Volume Road.

GENERAL LIMITATIONS

When a Low Cost Low Volume Road is referred to in this paper, the following limitations are used:

* Serves low volume traffic, less than 500 vehicles per day, and links areas within a regional coverage.

* Forms an access from and to rural areas, or scarcely-populated areas, or remote areas.

* Most of the route passes through non-residential areas.

* Pavement structures are composed of gravel with or without thin surface sealing.

Based on the limitations there are two interesting contradictive points to be considered; firstly, the road should be maintained in such a way so that it is always open to traffic which is normally from "under-developed" area to more-developed area, and vice versa; secondly, the pavement is in general composed of a marginal material which is very sensitive to climate but under a limited maintenance budget and other resources.

Compromising the two aspects could be a key to the success of optimising Low Cost Low Volume Roads as will be elaborated in the following sections.

MATERIAL

No doubt materials for Low Cost Low Volume Roads play an important role in satisfactory performance. Because of the importance of its role, there are many aspects to be considered in the selection of the material; for example: low cost and easy to use both for new construction and maintenance, easy to produce, and strong enough under the prevailing traffic. In other words, the material should satisfy the following requirements:

* Available locally or near the road in order to minimise transport cost.

* Low-grade or border-line quality. This can be examined by its grading and plasticity.

* Non-standard material, for example, lateritic soil, sandy material, silty sand and clayey soil, which can be stabilised with portland cement, lime or asphalt emulsion.

DESIGN

In the design process for Low Cost Low Volume Roads it is necessary to consider a statement of philosophy as follows:

"A design without possibility of failure would be unnecessary; and an allowance of 10% failure would probably be acceptable, taking into account the available budget".

The following criteria can be applied in the design of geometry, pavement structure and drainage of Low Cost Low Volume Road:

a. **Geometry**

- Maximum design speed: 60 km/hour
  Other components of geometry can be defined based on the design speed and ADT.
- High standard of horizontal alignment should not be applied although the space or land-use makes it possible. This is intended in order to maintain the maximum speed remains 60 km/hour, so that other components, for example, pavement type can be used effectively.

b. **Pavement**

- Generally the thickness of a pavement for Low Cost Low Volume Road need not be designed by the application of a normal procedure, for example, CBR-EAL Chart or Nomogram, but it is defined based on experience where 20-30 cm of gravel layer, with or without thin bituminous surfacing, is usually laid.
- Thin bituminous surfacing is suggested to use under the condition of as follows:
  + ADT of more than 200 vehicles/day. In this situation the headway between vehicles is very close so that the view in front of a rear vehicle will be obscured by dust if the pavement is not sealed. (see Attachment 1). In addition, when traffic of more than 200 vehicles/day travels over unsealed pavement, rate of gravel loss will be high.
  + Where the road passes through a populous area.
  + Where markets, schools, etc. are located near the road.
  + For specific sections, for example, steep grade section where the rate of gravel loss is high.

c. **Drainage Facilities**

- Under a limited budget surface drainage should be kept in good condition by provision of sufficient crossfall, such as 4% for unsealed road and 3% for sealed road.
- Unpaved side ditch should be well-maintained, especially in cut area.
- At steep grade sections of more than 7%, a thin paved side ditch should be utilised.

d. **Structure**

Any road, including Low Cost Low Volume Road, must be built across rivers, or pass through flooding areas. For high standard roads the problem can be resolved by a standard solution; that is, by erecting bridges or by building embankments to raise the level of the road surface.

For Low Cost Low Volume Roads implementation of standard solution will be very costly leading to high cost low volume road instead of low cost low volume road. In order to minimise the cost, the problem can be resolved by building "submerged bridges" at river crossings and building "submerged roadbeds" with protected side slopes at potential flooding areas. The construction of submerged bridges and submerged roadbeds can be done at areas where their topography is satisfied.

CONSTRUCTION AND SUPERVISION

Similar to other road construction, Low Cost Low Volume Roads built with the criteria mentioned above are sensitive to any variability, including variability during construction.

Because the variability, although small, will easily cause a reduction of the quality of the product it must be minimised. This can be done by intensifying the supervision of the works. From the viewpoint of Low Cost Low Volume Roads, the effort to intensify the supervision may be not a good solution, since it will increase the expenditure, hence a low cost effort must be created.

Three ways of reducing the variability in the provision of Low Cost Low Volume Roads are as follows:

a. Selecting a contractor with a good reputation, because a good reputation is the starting point of good quality.

b. Preparing an accurate and clear design as mentioned in Design, because a good design is also the starting point of good quality.

c. Supervising, in a simple way, for example, by focusing on key parameters like grading and plasticity.

MAINTENANCE

Any facility of relatively low initial cost will in general demand frequent maintenance at a relatively high cost. Any delay in maintaining facilities of low initial cost will significantly cause problems with the structure, in terms of fast deterioration of pavement components.

Efficient and effective maintenance of Low Cost Low Volume Roads can be done by the establishment of Units of Road Maintenance. The units should be based in strategic locations, be equipped with appropriate machines and their spare parts and be supported with a sufficient stockpiles of materials, for example, aggregate or gravel, bitumen, cement.

It is suggested that the strategic distance between locations of Maintenance Units (or Workshops) is not more than 50 km; whilst that of stockpiles is not more than 25 km. The frequency of the execution of the maintenance works by the maintenance units should be defined based on the condition of the road assessed by routine or periodic observations.

CONCLUSION AND RECOMMENDATION

Conclusions which can be drawn from this discussion can be summarised as follows:

a. The technology of Low Cost Low Volume Roads will remain a topical subject, because the existence of rural areas, remote areas and scarcely-populated areas will always be found throughout the world.

b. In order to achieve the Optimum Low Cost Low Volume Road, the main components and parameters which should be taken into account are as follows:

   - **Material**
     Local material (short hauling distance) to low grade or border line specifications with or without stabilisation can be used.

   - **Design**
     Geometric design should be based on a design speed of not more than 60 km/hour. Pavement design should be based on experience, that is, unsealed gravel road for road with ADT of 200 vehicles per day or less, and thin sealed surfacing for road with ADT of more than 200 vehicles per day or where populous areas, markets, schools and high gravel-loss sites are found.

     The thickness of the gravel layer is about 20 to 30 cm. Provision of high crossfall of pavement surface (3-4%), and thin paved side ditches at the high erosion potential sites. The construction of special submerged bridges, and submerged roadbeds at flooding areas should be considered.

   - Construction & Supervision
     Find a contractor of good reputation of contractor to carry out the works, because this is the starting point for high quality product.

     Keep supervision simple but concentrate on the key parameters of quality, such as, plasticity and grading.

   - **Maintenance**
     Provision of special task force Road Maintenance Units, based at strategic points, can increase the effectiveness and efficiency of the maintenance activities.

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ATTACHMENT 1

Maximum ADT Based on The Requirement of Sight Distance on The Dusty Unsealed Gravel Road

Assumption

Minimum sight distance due to visibility in dusty condition
= the safe headway of vehicle in dusty condition
= 300 m

$$\text{Headway} = \frac{\text{Average speed of vehicle in meter/minute}}{\text{Average number of vehicle per minute (DMV)}}$$

Average speed = 60 km/hour = 60,000 m/60 minutes
= 1,000 m/minute

Average number of vehicle per hour (DHV) = 13.0% ADT

Average number of vehicle per minute (DMV) = 1.67% ADT

$$\text{Headway} = \frac{\text{Average speed of vehicle in meter/minute}}{\text{Average number of vehicle per minute}}$$

$$300 \text{ m} = \frac{1{,}000}{1.67\% \text{ ADT}}$$

$$\text{ADT} = \frac{1{,}000}{1.67\% * 300} = 199.6, \text{ rounded to } 200$$

Maximum ADT for dusty gravel road = 200 vehicle/day

THE PERFORMANCE OF CEMENT-STABILISED ROAD BASE IN UPGRADING A RURAL ROAD IN MALAYSIA

by

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ABSTRACT

This paper describes the construction and assesses the performance of a soil-cement-additive road base in upgrading a rural road in Malaysia. After almost two years in service, its performance is comparable with the control section which comprises of a much thicker construction.

INTRODUCTION

In Malaysia most laterite roads are upgraded by the construction of a subbase and crushed aggregate road base which would then be sealed with surface dressing or asphaltic concrete. Due to the high cost of transporting quarry materials to the construction site, especially in the southern part of Peninsular Malaysia, a research project was conducted in a land development scheme (FELDA). At this site, soil-cement with additive was used as road base and asphaltic concrete was used as surfacing. The soil-cement included an additive which was claimed by the manufacturer to enhance the strength of the stabilised soil and also to prevent the pavement from cracking at an early stage of construction. The performance of the soil-cement-additive road base was monitored and compared with that of the crushed aggregate road base in terms of its crack and rut characteristics.

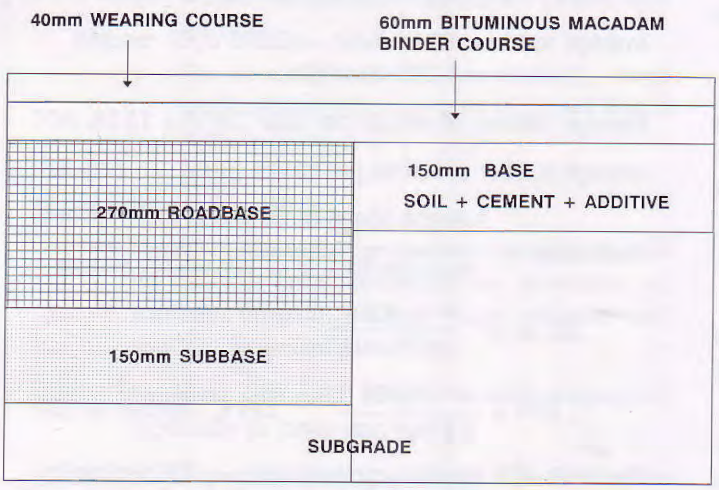

Figure 1: Profile of the Pavements

RESEARCH SITE

The research site is situated at FELDA Palong Timor 3, Segamat, Johor, in the south of Peninsular Malaysia. It is an access road to a palm oil mill. The project involved the construction of a new road leading to a newly constructed palm oil factory. When the factory started its operation, it was expected that heavy loaded lorries would use this road.

The length of the road was only 600m long and comprised of fill and cut sections. The research stretch from chainage 50 to chainage 250 ran through cut and fill sections. The construction was completed in May 1992.

DESIGN

The road was designed to Public Works Department Malaysia R1 standard with a pavement width of 5.0m.

The structural design of the pavement sections is shown in Figure 1. The soil-cement- additive mix was constructed to a thickness of 150mm without sub-base. For other stretches, the pavement was designed with a 150mm thick sub-base and 270mm crushed aggregate road base. For surfacing 90mm asphaltic concrete was laid for the whole length of the road.

The cement mixture was approximately 3 percent additive and 97 percent cement, by weight. The amount of the cement mixture added to the soil was about 8.5 percent by weight. For the 150mm thick pavement layer to be constructed, 6 kg of additive with 200 kg of cement was laid on 1 m^2 pavement.

CONSTRUCTION

Imported laterite was used for subgrade and compacted to at least 95 per cent of the maximum dry density. After compaction, the surface was levelled to the required formation level. The subgrade was then scarified to the required depth by means of a rotary machine to ensure that the maximum particle size of the soil was less than 30mm, as required in the specification. Soils with particle size greater than 30 mm would be relatively difficult to stabilise and may not achieve maximum stabilisation. A few passes of the rotary machine was required before the pulverisation process was completed. The pulverised soil was then levelled again using a farm tractor.

Ordinary portland cement and the powder additive were mixed in a concrete mixer. After mixing, the cement-additive mixture was spread evenly on top of the pulverised soil using pails or buckets to cover the predetermined area. The mixing was done in a dry state.

The rotary machine was again used to mix the cement-additive and soil until the colour became uniform. Water was sprayed onto the mixed materials and mixing by the rotary machine was then continued until the desired mixture was obtained. During the mixing process, water was added to the mixture up to the optimum moisture content required for compaction.

Immediately after the mixing work was completed a 10 ton roller was used to compact the mixed materials until it reached at least 95% of Maximum Dry Density (Modified). Soon after compaction, the surface was trimmed to camber and superelevation by a motor-grader. The pavement cured for several days spraying water regularly.

After about 20 days, the bituminous surfacing was laid.

PROBLEMS DURING CONSTRUCTION

Several problems were encountered during the construction of the soil-cement-additive road base:

* Emission of cement and additive dust to the air was very great during mixing work which was done in concrete mixers and also during pulverisation process.

* Rain during or after the pulverisation process spoiled certain section of the test section which had to be repaired. It was therefore necessary to take into consideration the weather condition before starting work.

* Strict control and supervision was needed when spreading the cement-additive mixture.

* The amount of water required was difficult to control because the condition of the mixture at the bottom could not be accurately ascertained. On this matter the decision on whether the moisture content was adequate or not was based solely on experience.

MATERIAL PROPERTIES

Subgrade Strength

Prior to the construction of the pavement, the structural strength of the subgrade in term of its in situ California Bearing Ratio (CBR) values was determined using the Dynamic Cone Penetrometer test method.

The results show that the strength of the subgrade at the trial section using soil-cement-additive was between CBR 8 to 17 percent in situ and that using the crushed road base was between CBR 9 and 17 percent in situ.

The Properties of Soil for Stabilisation

The soil used in the soil-cement-additive road base and was classified as A-6(8) and A-7-5(7) soil with optimum moisture content of 18 percent. It had a mean soaked CBR strength of 13 percent and a mean unsoaked CBR of 3.5 percent.

FAILURE CRITERIA AND RESULTS

The failure criteria adopted were those that could be directly measured on the pavement surfacing, and they are:

* Mean rut depth exceeding 15mm

* Presence of interconnected cracks

For monitoring purposes, the test sections were divided into 10-metre test blocks and were assessed at about 3-monthly intervals.

After 21 months, both sections had performed well with mean rut depths of about 3mm (Figures 2 and 3). None of the test blocks, either using soil-cement-additive road base or crushed stone road base had experienced interconnected cracks.

CONCLUSIONS

a. A lot of problems were encountered during the construction of the trial pavement, such as emission of cement particles to the air, problem

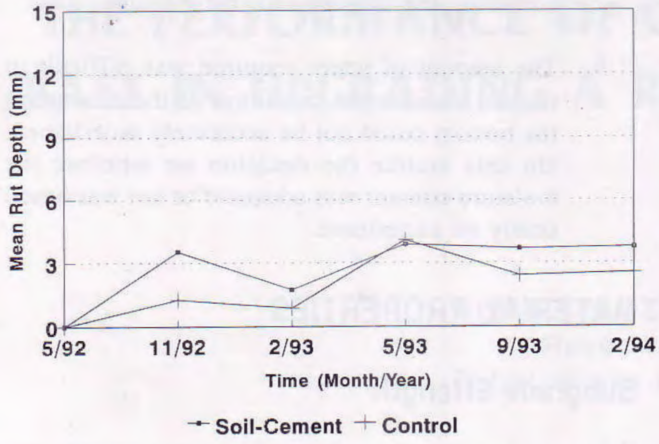

Figure 2: Rutting Measurement Survey
Felda Palong Timur 3, Segamat, Johor
To Palm Oil Mill

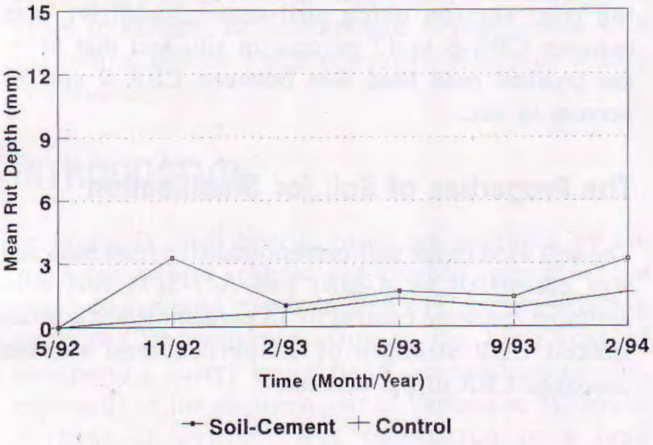

Figure 3: Rutting Measurement Survey
Felda Palong Timur 3, Segamat, Johor
From Palm Oil Mill

in handling the rotary machine, the weather and mixing. All these problems must be taken into consideration before embarking on the method.

b. Despite being much thinner and the use of cheaper materials, the soil-cement-additive road base showed comparable performance to the crushed aggregate road base after 21 months in service.

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CONFERENCE ON ASIAN ROAD SAFETY

25-28 OCTOBER 1993

THE KUALA LUMPUR AGENDA

The Kuala Lumpur Agenda, was published in the January 1994 issue of the Journal. OECD has informed us of some important modifications, arising from comments received. The finalised Agenda is now published once again for the benefit of all REAAA members and readers

Road accidents are a major problem in the Asian Region. The rapidly growing economies and increasing motorisation will inevitably lead to a deteriorating road safety situation. Opportunities exist to intervene and prevent unnecessary deaths and disabilities and the resulting economic losses. The following recommendations derived from the above Conference are intended to provide guidance for countries seeking to tackle their road safety problem.

Road Safety Policy and Research in Asian Countries

1. Strategic planning for road safety is important and this should be related to transport systems and land use policies.

2. Targetted safety programmes should be identified systematically. Prioritised short and medium-term accident countermeasure programmes need to be developed. This should include proper assessment before implementation, monitoring during implementation and evaluation on completion.

3. The establishment of effective organisational and management structures for the development and implementation of road safety policies and measures at national, regional and local levels is strongly recommended. Coordination of all relevant agencies, including private and voluntary organisations is essential to the success of the programmes. Committment at the highest political level is crucial to success.

4. Adequate public funding of road safety activities needs to be assured. In some countries good results have been achieved by earmarked safety funds. Opportunities for supplementary funding through private sponsorship or sources, for example, insurance, should be explored. Funding for road safety is increasingly available from development banks, for example, World Bank and ADB and bilateral agencies. Countries are encouraged to tap into such funds.

5. In order to optimise the impact of the limited resources available for road safety improvements, it is essential that an assessment be made of the costs of different types of accidents. Countries should initiate investigations on costs of road accidents so that road safety countermeasures and action plans can be systematically assessed for cost effectiveness.

6. No safety programme can be successfully developed and put into practice without in-depth safety expertise and know-how at the front line and at proper decision levels within the authority concerned. Above all, correct implementation of safety actions is necessary.

7. Continous upkeep of safety expertise in key executive and management positions and/or the hiring of recognised outside safety specialists are important for effective and professional operations.

8. The development of local road safety specialist and researchers is essential to ensure sustainability of effective road safety programmes. This can be achieved through in-country practical training, study tours and international cooperation.

9. Research is a vital element of effective safety programmes. Countries are encouraged to collaborate and to initiate research programmes on appropriate issues. The establishment of a regional and/or national road safety research institute should be considered.

10. Public awareness of safety issues and requirements should be raised through the most appropriate measures, such as information campaigns. Education programmes for target groups, for example, children, should be developed and implemented.

11. Effective systems for accident data collection and analysis are essential for problem identification and the development and

assessment of countermeasures. Countries with inadequate systems are encouraged to improve them; and those that already have good systems are encourage to collaborate and harmonise to facilitate comparisons. A joint evaluation of existing systems and tools, through the most competent international organisation in Asia is recommended.

12. Safety of vulnerable road users should be accorded the highest priority in the development of countermeasures. The specific transport situation in Asian countries requires focused safety development and research for paratransit and truck safety (i.e. overloading).

13. Research and cost effective countermeasures should focus on the specific needs of Asian countries and such informatioon should be disseminated through appropriate manuals (especially on low-cost engineering counter measures), guidelines and training courses.

14. Regional cooperation is required in a number of sectors related to road safety. Initially, the focus should be on compilation and review of current traffic legislation and regulations.

15. The training of safety professionals is essential to ensure effective programme management. Comprehensive courses for trainers in the Asian region should be implemented through REAAA, with financial assistance from ADB. A road safety journal for Asia should be launched.

16. Technology transfer in the road safety field should be enhanced through appropriate regional and international networks, for example, PIARC (Permanent International Association of Road Congress) and through technical co-operation and exchange of experience among developing countries. The OECD's IRTAD (International Road Traffic and Accident Database) – providing harmonised cross country comparisons through aggregate accident data for 29 countries – is open for participation by Asian countries.

Second Conference

A Second Conference on Asian Road Safety to be held in 1996, preferably in the People's Republic of China is proposed.

Its objectives will be:–

* to follow up the results of the above initiatives
* to monitor the changes in the traffic safety situation in Asian countries
* to update safety professionals on the latest developments.